

DESIGN OF
OPEN SPANDREL REINFORCED
CONCRETE ARCH BRIDGE

BY
R. J. JENSEN
E. O. MANDLER
C. H. MARX

ARMOUR INSTITUTE OF TECHNOLOGY

1911

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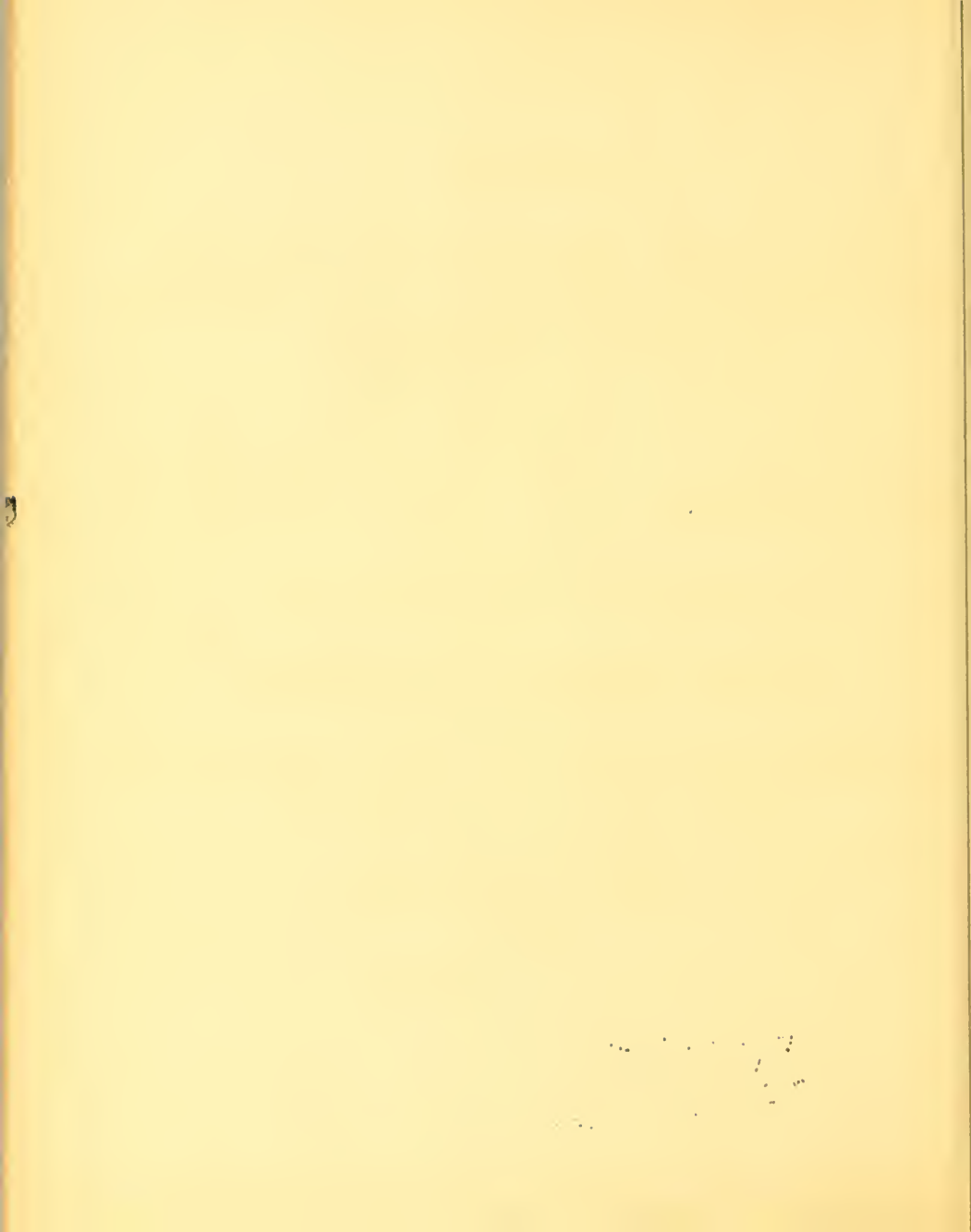


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Design of an open spandrel
reinforced concrete arch



DESIGN

Of an Open Spandrel Reinforced Concrete Arch Bridge of Two Hundred and Ten Feet Span.

A Thesis

PRESENTED BY

RAYMOND F. JENSEN

EMIL O. MANDLER ✕

CARL H. MARX

TO THE

PRESIDENT AND FACULTY

OF

Armour Institute of Technology

FOR THE DEGREE OF

BACHELOR OF SCIENCE IN CIVIL ENGINEERING

HAVING COMPLETED THE PRESCRIBED

COURSE OF STUDY IN

Civil Engineering

1911.

Approved:

Wm. E. Phillips
Prof. of Civil Engineering

Sam. Raymond
Dean of Eng. Studies
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Dean of Cultural Studies

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DESIGN OF AN OPEN SPANDREL REINFORCED
CONCRETE BRIDGE WITH SOLID RIB AND
DIAPHRAGMS - 210' SPAN AND 44' RISE.

-METHOD OF DESIGN-

The method used in the design and analysis this arch was that given by Turneaure and Maurer in their treatise on "Principles of Reinforced Concrete." This application of the theory of the arch was followed throughout.

-THE ARCH RIB-

In the design of any bridge, certain assumptions must be made; this fact being more manifest in the case of a concrete or masonry arch rib. It is the usual custom to assume a preliminary design made by the aid of approximate or empirical rules or by reference to the proportions of existing arches. This arch is then analyzed and the results used in correcting the design, the corrected design may then in turn be analyzed, if it departs too greatly from the first assumed.

REPORT OF THE JOINT CHIEFS OF STAFF
ON THE MATTER OF THE
ATTACK ON THE 11th SEPTEMBER 1941.

1. SUMMARY.

The attack on the 11th September 1941 was a surprise attack on the British fleet in the Atlantic. The attack was carried out by a force of German submarines and aircraft. The attack was successful in sinking the British fleet and in capturing the British fleet. The attack was a major disaster for the British fleet and for the British Empire.

2. DETAILS.

The attack on the 11th September 1941 was a surprise attack on the British fleet in the Atlantic. The attack was carried out by a force of German submarines and aircraft. The attack was successful in sinking the British fleet and in capturing the British fleet. The attack was a major disaster for the British fleet and for the British Empire.

To begin with we agreed to make the arch rib solid instead of using separate arch rings, as has been the common practise in most concrete bridges and viaducts. The spandrel were also made solid. These assumptions simplify the design somewhat as the dead load and live load were concentrated uniformly over the rib. The thickness of the rib at the crown was assumed 3 ft. and at the haunch as 5 ft. The roadway is supported on spandrel walls 18 inches thick resting upon the arch rib. The spacing of the diaphragms was assumed as 15 ft. making the distance between springing lines 210 ft. The rise of the arch at the crown was taken as 44'0". The spandrel walls at the crown is 5'0" from the axis of the arch to the underside of the floor beam. The arch was designed with .5% of steel reinforcement above and below the axis.

NOTATION- (See Plate 1.)

Let H_o = thrust at the crown;

V_o = shear at the crown;

M_o = bending moment at the crown, assumed as
positive when causing compression in
the upper fibres;

$N, V, \& M$ = thrust, shear, and moment at any section;

R = Resultant pressure at any section
resultant of N and V ;

ds = length of a division of the arch ring
measured along the arch axis;

n = number of divisions in one half of the
arch;

I_a = moment of inertia of any section;

P = any load on the arch;

x, y = co-ordinates of any point on the arch
axis referred to the crown as origin,
and all to be considered as positive
insign;

m = bending moment at any point in the canti-
lever, due to external loads.

1. The first part of the document is a list of names and addresses, including "Mr. J. H. Smith, 123 Main St., New York, N. Y." and "Mr. J. H. Smith, 123 Main St., New York, N. Y."

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Theoretically the gain in economy by the use of steel in the concrete arch is not great. If the pressure line is not depart from the middle third, the steel reinforces only in compression and in this respect is not as economical as concrete. If the line of pressure deviates farther from the center, resulting in tensile tresses in the steel, the conditions are such that these stresses must be provided for by use of the steel at very low working values. That is to say, the direct compression in the arch is so large a factor that the limiting stresses in the concrete will always result in very small unit tensile stresses in the steel where any tension exists at all.

Practically the value of reinforcement is very considerate. It renders an arch of much more secure and reliable structure, it greatly aids in preventing cracks due to any slight settlement, and by furnishing a form of construction of greater reliability makes possible the use of working stresses

A. The following is a list of the names of the persons who have been elected to the office of the President of the United States, and the names of the persons who have been elected to the office of the Vice President of the United States, in the year 1800.

1. John Adams, President of the United States.

2. Thomas Jefferson, Vice President of the United States.

3. George Clinton, Vice President of the United States.

4. Aaron Burr, Vice President of the United States.

5. James M. Smith, Vice President of the United States.

6. John Jay, Vice President of the United States.

7. John Rutledge, Vice President of the United States.

8. John C. Calhoun, Vice President of the United States.

9. John Pickens, Vice President of the United States.

10. John M. Pickens, Vice President of the United States.

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In the concrete considerably higher than is usual in plain masonry. Furthermore, in long spans such as ours, where the dead load constitutes by far the larger part of the load, any possible increase in average working stress counts greatly toward economy. It affects not only the arch but the abutment and foundation.

The roadway was made 30'0" from curb to curb, leaving room for two tracks for an electric railway. The roadway is to be paved with asphalt having a two inch cushion of sand. The sidewalks are 10'0" wide supported by cantilever brackets. The sidewalk is to be furnished with a concrete railing having a post at each panel point. An electric lamp is to be placed at every other post.

-DESIGN-

The analysis of any arch consists in the determination of forces acting at any section usually expressed as the thrust, the shear and the bending moment at that point.

The thrust we use, was taken to be the component of the resultant parallel to the arch, axis at the given point and the shear is the component at right angles to such axis. The thrust causes simple compressive stresses; the shear causes stresses similar to those produced by the vertical shear in a simple beam. The method of procedure will be to determine first, the thrust, shear, and bending moment at the crown. These being known, the values of similar quantities for any other section can readily be calculated.

Before any of the above mentioned values can be determined the arch ring has to be divided into preliminary and final divisions, the central points of which we investigate for shear etc. In most cases the depth of the arch ring increases from crown towards springing line giving a variable moment of inertia. Considering the concrete only the moment of inertia will increase as d so that comparatively small change in depth will cause a large change in moment of inertia.

The amount of work, which is to be done, is
presented in the following table in the order,
as the first, second and third, in the sequence
of the engine to work. The total amount
of the compressive force; the power can be
as shown in the following table. The
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To maintain ds/I constant, the value of ds will therefore be much greater near the springing line than at the crown and hence to secure the desired accuracy the length of division at the crown will need to be made fairly short. The value of ds/I to adopt so that there will be no fractional division is: $ds/I = S/n$ where I is mean value of moment of inertia.

S is half length of the arch ring measured along the axis.

n is number of divisions in one half of the arch.

First we calculated the mean value of i for each division of half the arch (see plate "A") after we had scaled the depths at the mid-points of each division. Knowing the amount of steel in the arch we figured the moment of inertia of it about the same axis. To get the total moment of inertia " I " we multiplied this value by 15 and added to it the I (Plate "A".) $I = I_a + 15 I_s$

THE UNITED STATES OF AMERICA
DO hereby certify that
the following is a true and correct
copy of the original as the same
exists in the records of the
Department of the Interior.
Witness my hand and seal of office
this 1st day of July, 1901.
J. H. ROBERTSON, Secretary of the Interior.
By _____, Assistant Secretary.
This is to certify that the
above is a true and correct
copy of the original as the same
exists in the records of the
Department of the Interior.
J. H. ROBERTSON, Secretary of the Interior.
By _____, Assistant Secretary.

The average $\bar{i} = i/n = 3.4769/14$

The value of ds_i being known, the proper length of ds for any part of the arch ring can readily be determined. The half length of the arch axis was found to be 116.97 feet. The first part of the table A relates to the preliminary 14 equal divisions. Each equal to $116.97/10 = 11.697$ ft. The resulting values of i were plotted as shown in plate-3. The line ab is 116.97 feet long and was divided into 14 equal divisions as 1,2,3,etc. At the center of the several divisions the values of small i were laid off as ordinates $i_1 i_2 i_3$ etc., and the curve cd was drawn through these points.

The area $abcd = 116.97/14 \times \sum i = 116.97 \times i_a$
 This area is to be divided into fourteen equal parts each equal to ds_1 . Each of these parts will then be equal to $116.97 \times i_a / 14 = 2.074$ as given below table A. Beginning at one end of this diagram the several equal areas are then laid off,

[illegible]

the values of i being scaled from the diagram and ds is equal to $2.074/i$. These calculations are given in the latter part of table A, where are also given the values of I and d for the center points of the final subdivisions.

To obtain the thrust, shear and bending moment at the crown we used the formulae:

$$H_o = \frac{n \sum my - \sum m \sum y}{2[(\sum y)^2 - n \sum y^2]}$$

$$V_o = \frac{\sum (m_R - m_L)x}{2 \sum x^2}$$

$$M_o = \frac{\sum m + 2H_o \sum y}{2n}$$

In these equations the summations $\sum y$, $\sum y^2$ and $\sum x^2$ are for one-half of the arch only; the summations $\sum m$ is for the entire arch and is equal to $\sum m_R + \sum m_L$; the summation $(m_R - m_L)x$ is a summation of the products $\sum (m_R - m_L)x$, in which

m_R and m_L are the bending moments at corresponding points in the right and left halves which have equal abscissas x ; and the summation $\sum my$ is for the entire arch, but since symmetrical points have equal y 's this quantity may be calculated as $\sum (m_R + m_L)y$.

In designing an arch it is sufficient generally to determine the maximum stresses at the crown, the haunch, and the springing line. This will require several different positions of the live-load. For the crown the maximum positive moments are caused when a short length of the arch at the center (middle third) is loaded, and the maximum negative when the remaining portions are loaded. The maximum positive and negative moments at the haunch (about the $1/4$ point) are caused when the whole span length is loaded. A condition was also taken with the half span length loaded.

The values of x and y in these equations, were accurately scaled from the drawing. The values of m and m were figured for the different loadings and their summation taken as shown in tables BC&D. A simple substitution in equations for H_0 , V_0 , and M_0 gave us the values for each of the three conditions of loading.

All the loads in this design were vertical, so that the graphical method might easily have been to advantage in determining the cantilever moments "m."

With these values calculated we lay out the force polygons using H as the pole distance and M/H as the distance above or below the axis. The equilibrium polygon was indrawn (plate C-2.) and the eccentric distances obtained. The thrust was measured or scaled directly from the true force polygon.

The total bending moment at any section, 1,2, 3, etc. was found from the equation :

$$M = m + M_o + H_o y \pm V_o x$$

The plus sign was used for the left half and the minus sign for the right half of the arch. Knowing the moment and thrust (Table E,) at each point the eccentric distances were found since $e = M/H$. If calculations are to be made for more than one loading it will be noted that the denominators of the values for H_o , V_o , and M_o , do not change.

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Having the values of the bending moments and eccentricities at each of the fourteen points, the next step was to find the unit stresses in the concrete and steel. To calculate these stresses we used the formulae for simple beams:

$$\frac{M}{bh^2 f_c} = \frac{1}{12k} (1 + 24np\bar{a}/h)$$

To facilitate the application, of this equation,

Plates XIII and XIV pages 287 and 288 in Turneaure and Maurer² were used. Knowing the eccentricity and depth of the beam, a simple division gave us e/h . In the first diagram, values of the eccentricities, e/h , are given at the upper and lower margins; the ordinates from the lower margins to any curve are values of $(1 + 24np\bar{a}/h)/12k$, and hence of $M/bh^2 f_c$, for the values p marked on that curve.

For instance take point nine on plate c

$$e/h = .353/3.78 = .0746 \quad \text{where } p = .5\%$$

$$M / bh f_c = 0.0635 \quad \text{but } M = 46.4$$

$$\therefore f_c = 434 \times 12 / 3.78 \times 12 \times 144 = 464 \text{ psi sq. in.}$$

In this manner we figured the unit stress in the concrete for each point in the arch under the three loadings. This stress occurs in the upper fibres of the arch, while the value in the lower fibres is equal to :

$$f'_c = f_c (1 - 1/k) \text{ which is always less than } f_c$$

The stresses in the steel were calculated from equations :

$$f'_s = n f_c (1 - d'/kh)$$

$$f_s = n f_c (1 - d/kh)$$

With the value of e/h we found the value of $1/k$ from figure 33, page 103 Turn. and Maurer.

By simple substitution and a little mathematics we obtained the stresses in the steel which as is shown in plate c were safe. Since all the stresses in the concrete and steel were under the allowable values of $f_c = 600$ and $f_s = 16000$ the arch is safe.

-TEMPERATURE STRESSES!-

The temperature stresses were obtained by means of the equation:

$$H_o = \frac{EI}{ds} \times \frac{ct \ln}{2[n \Sigma y - (\Sigma \bar{y})^2]}$$

where H is the thrust at the crown produced by the restraint of the abutment.

c = coefficient of expansion = .0000054

l = span 210'.

t = temperature in degrees = F = 30

E = coefficient of elasticity 1,500,000#/in.

I = moment of inertia ds/I 3.1

$$H_o = \frac{150000000 \times 144}{3.1} \times \frac{.0000054 \times 30 \times 210 \times 14}{2[14(2913.85) - 129.61]}$$

$$H_o = \frac{103000000}{148600} = 693.0\#$$

$$M_o = -693 \times 129.6 / 14 = -6,418.9 \text{ ft. lbs.}$$

The equilibrium polygon is a horizontal line drawn a distance below the crown equal to $6418.9 / 693 = 9.26$ ft. The moment at any point is equal to the thrust H_o multiplied by the vertical distance from such point to the equilibrium polygon.

1. The first step is to determine the temperature of the system.

2. The second step is to determine the pressure of the system.

$$P = \frac{nRT}{V}$$

3. The third step is to determine the volume of the system.

4. The fourth step is to determine the number of moles of the system.

5. The fifth step is to determine the gas constant.

6. The sixth step is to determine the temperature of the system.

7. The seventh step is to determine the pressure of the system.

8. The eighth step is to determine the volume of the system.

9. The ninth step is to determine the number of moles of the system.

$$P = \frac{nRT}{V}$$

$$P = \frac{nRT}{V}$$

10. The tenth step is to determine the temperature of the system.

11. The eleventh step is to determine the pressure of the system.

12. The twelfth step is to determine the volume of the system.

13. The thirteenth step is to determine the number of moles of the system.

14. The fourteenth step is to determine the gas constant.

15. The fifteenth step is to determine the temperature of the system.

Temperature

Expansion joints in the concrete were allowed every 50 ft. and consisted of a few sheets of tar paper inserted in the joint. When not reinforced concrete will, under such circumstances crack at intervals, its maximum deformation under stresses not being equal to its maximum temperature deformations. It is to be assumed that concrete when reinforced will not stretch more than plain concrete, as seems probable, then no amount of reinforcement can entirely prevent contraction cracks. The reinforcement can entirely, however, force such cracks to take place as they do in a beam,--at such frequent intervals that the requisite deformation takes place without any one crack becoming large. These temperature stresses obtained were very safe and are entirely taken up by the steel in the arch.

1844-1845. The following table shows the number of
persons who were admitted to the hospital during the
year 1844-1845. The total number of admissions was
1,234. The number of persons who were discharged was
1,098. The number of persons who died was 136.
The number of persons who were cured was 1,062.
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1844-1845.

-LOADING-

Dead Load-

Concrete, including reinforcement, = $150\frac{\#}{\text{cubic foot}}$

Asphalt pavement, including 6" concrete foundation, filling of gravel under pavement, also street car construction, complete is $140\frac{\#}{\text{cubic foot}}$

Live Load-

Street car =	35 tons	} Chicago Bridge Dept. (See fig. A, plate 1.)
Sprinkler =	42 "	

Uniform load of $100\frac{\#}{\text{square foot}}$ over the rest of bridge roadway. Sidewalk load equals $60\frac{\#}{\text{square foot}}$.

-PANEL LOADS-

Live Load-

(See fig. B, plate 1.) Car and sprinkler are shown as regards lengths over diaphragm. We are considering them as placed side by side so as to obtain the greatest weight possible over diaphragm. (Note ! sketch does not show them side by side.)

We therefore, have over the diaphragm approximately one half of each, that is, $42/2 + 35/2 + = 77/2$ tons. This (77/2 tons) is considered as distributed evenly over the (33 ft.) width of the diaphragm.

$$77/2 \times 2200\# = 84700\# = 84.7 \text{ kips.}$$

Remaining roadway of 14 feet $= 100\#$ per sq. foot. $100\# \times 14 \times 15 = 21 \text{ kips.}$

$$\text{Sidewalk} = 60\# / \text{sq. ft. } 60 \times 2 \times 10 \times 15 = 18 \text{ kips.}$$

$$\text{Total} = 84.7 + 21 + 18 = 123.7 \text{ kips.}$$

This is evenly distributed over 33 foot diaphragm. (For calculations we consider diaphragm as 12" wide across bridge.

\therefore Live panel load $= 123.7 / 33 = 3.75 \text{ kips per ft.}$
width of bridge.

-DEAD PANEL LOADg-

Note! (See fig.C, plate 1.) Dead panel load takes in material from lines AA to BB.

$$P, \quad y = 36' \quad (\text{fig.D, plate 1.})$$

Assume diaphragm as 18" thick = 1 foot-6"

$$\text{Its volume} = 36' \times 15/10 \times 1' = 54 \text{ cu.ft.}$$

$$54 \times 150\# = 8.1 \text{ kips.}$$

Distance ϕ , of arch rib is 17 feet.

Thickness = 4 1/2 feet. Width taken as 1 ft.

Volume = $17 \times 4 \frac{1}{2} \times 1 = 77$ cubic feet.

$$77 \times 150 \# = 11.55 \text{ kips.}$$

Roadway floor-slab assumed as 8" deep = 2/3 feet.

Weight per foot of length across the bridge is

$$15 \times 1 \times \frac{2}{3} \times 150 \# = 1.5 \text{ kips}$$

Sidewalk assumed as 6" deep.

$(2 \times 15 \times 10 \times \frac{1}{2} \times 150 \#) \div 335 = 335.75 \text{ k. per foot width of bridge.}$

8" pavement,; consisting of asphalt etc. Weight per ft. width of bridge is $140 \times 15 \times \frac{2}{3} = 1.4$ kips

Total dead load =

$$8.1 + 11.55 + 1.5 + .75 + 1.4 = 23.30 \text{ kips}$$

Total Live load

$$= 3.75 \text{ kips}$$

$$\text{Total} = P_1 = 27.05 \text{ kips}$$

P_2

$$y_2 = 26' \quad 26 \times 1 \frac{1}{2} \times 1 = 39 \text{ cu. ft.} \quad a_2 = 17'$$

Thickness = 4 1/4 feet $39 \times 150 \# = 5.85 \text{ k}$

$17 \times 4 \frac{1}{4} \times 1 \times 150 \# = 10.95 \text{ kips}$

Floor slab = 1.5 K.

Pavement = 1.4 K.

Wavelength = 4.10 microns. Wavelength = 4.10 microns.

Intensity = 1.00 microns. Intensity = 1.00 microns.

Wavelength = 4.10 microns. Wavelength = 4.10 microns.

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-P₄-

$$y_4 = 12' \quad 12 \times 1 \frac{1}{2} \times 1 = 18 \text{ cu. ft.}$$

$$a_4 = 16.5 \times 3.5 \times 1 \times 150\# = 8.7 \text{ K.}$$

$$18 \times 150\# = 2.7 \text{ K.}$$

$$\text{Sidewalk, floor, pavement.,} = 3.65 \text{ K.}$$

$$\text{D. L.} = 15.05 \text{ K.}$$

$$\text{L. L.} = \underline{3.75 \text{ K.}}$$

$$\text{Total} = 18.8 \text{ K.} = \text{panel load } P_4.$$

-P₅-

$$y_5 = 8 \text{ feet.} \quad 8 \times 1 \frac{1}{2} \times 1 \times 150\# = 1.8 \text{ K.}$$

$$a_5 = 15.5 \times 3.25 \text{ ft.} \quad 8 \times 1 \frac{1}{2} \times 1 \times 150\# = 1.8 \text{ K.}$$

$$15.5 \times 3.25 \times 1 \times 150\# = 7.65 \text{ K.}$$

$$\text{Sidewalk., etc.,} \quad 3.65 \text{ K.} \quad \text{D. L.} = 13.1 \text{ K.}$$

$$\text{L. L.} = \underline{3.75 \text{ K.}}$$

$$\text{Panel load } P_5. \quad \text{Total} = 16.85 \text{ K.}$$

-P₆-

$$y_6 = 6' \quad 6 \times 1 \frac{1}{2} \times 1 \times 150 = 1.35 \text{ K.}$$

$$a_6 = 15 \times 3 \quad 15 \times 3 \times 1 \times 150\# = 6.75 \text{ K.}$$

$$\text{Sidewalk, pavement, etc.,} = 3.65 \text{ K.}$$

$$\text{D. L.} = 11.1 \text{ K.}$$

$$\text{L. L.} = \underline{3.75 \text{ K.}}$$

$$\text{Total} = 15.5 \text{ kips panel load } P_6.$$

-2-

1. 10. 1901 - 1. 10. 1902
2. 1. 1902 - 1. 1. 1903
3. 1. 1903 - 1. 1. 1904
4. 1. 1904 - 1. 1. 1905
5. 1. 1905 - 1. 1. 1906
6. 1. 1906 - 1. 1. 1907
7. 1. 1907 - 1. 1. 1908
8. 1. 1908 - 1. 1. 1909
9. 1. 1909 - 1. 1. 1910
10. 1. 1910 - 1. 1. 1911

-3-

1. 1. 1911 - 1. 1. 1912
2. 1. 1912 - 1. 1. 1913
3. 1. 1913 - 1. 1. 1914
4. 1. 1914 - 1. 1. 1915
5. 1. 1915 - 1. 1. 1916
6. 1. 1916 - 1. 1. 1917
7. 1. 1917 - 1. 1. 1918
8. 1. 1918 - 1. 1. 1919
9. 1. 1919 - 1. 1. 1920
10. 1. 1920 - 1. 1. 1921

-4-

1. 1. 1921 - 1. 1. 1922
2. 1. 1922 - 1. 1. 1923
3. 1. 1923 - 1. 1. 1924
4. 1. 1924 - 1. 1. 1925
5. 1. 1925 - 1. 1. 1926
6. 1. 1926 - 1. 1. 1927
7. 1. 1927 - 1. 1. 1928
8. 1. 1928 - 1. 1. 1929
9. 1. 1929 - 1. 1. 1930
10. 1. 1930 - 1. 1. 1931

Total dead load equals

$$5.85 + 10.95 + 1.5 + .75 + 1.4 = 20.45$$

Live load equals 3.75

$$\text{Total Panel load } P_2 = 3.75 + 20.45 = 24.20 \text{ K.}$$

- P_3 -

$$y_3 = 18' \quad 18 \times 1 \frac{1}{2} \times 1 = 27 \text{ cu. ft.}$$

$$a_3 = 16.5 \text{ ft.} \quad \text{thickness is } 3.75 \text{ ft.}$$

$$16.5 \times 3.75 \times 1 \times 150\# = 9.285 \text{ k.} \quad 27 \times 150\# = 4.05 \text{ k.}$$

$$\text{Floor, sidewalk, pavement same as above} = 3.65 \text{ K.}$$

$$\text{D. L.} = 16.985 \text{ K.}$$

$$\text{L. L.} = \underline{3.750 \text{ K}}$$

$$\text{Total} = 20.74 \text{ K. panel load } P_3$$

- P_7 -

$$y_7 = 5 \text{ ft.} \quad 5 \times 1 \frac{1}{2} \times 1 = 7.5 \text{ cu. ft.}$$

$$a_7 = 14.5 \times 3 \quad 14.5 \times 3 \times 1 \times 150\# = 6.525 \text{ K}$$

$$7.5 \times 150\# = 1.125 \text{ K.}$$

$$\text{Floor, sidewalk, and pavement} = 3.65 \text{ K.}$$

$$\text{D. L.} = 11.30 \text{ K.}$$

$$\text{L. L.} = \underline{3.75 \text{ K}}$$

$$\text{Total} = 15.05 \text{ K. panel load } P_7$$

DESIGN OF SIDEWALK AND FLOOR SYSTEM.

The floor system consists of reinforced concrete slabs resting on floor longitudinal girders or stringers, spaced as shown. The two inside stringers are spaced 10'-0" c.to c., being directly under the center lines of the tracks. The sidewalk slabs are supported by the outside stringers and by beams caeeiwd on the ends of cantilevers placed every 15' (Fig.B. Plate-11.)

-THE SIDEWALK-

Live load on sidewalk = 60# per square foot. The width of the sidewalk, and hence the span of the the slab, is taken as 10'-0". From Turneaure and Maurer Reinforced Concrete Construction. Table 21,(7) page 298, we find that for this span and loading a 6" slab may be used, for a value of the bending moment of $1/12 Wl$. The required area of steel per foot of width for this slab is .385 sq. in. This will be furnished by steel rods.

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7. 1998. 2000. 2002. 2004. 2006. 2008. 2010. 2012. 2014. 2016. 2018. 2020. 2022. 2024. 2026. 2028. 2030. 2032. 2034. 2036. 2038. 2040. 2042. 2044. 2046. 2048. 2050. 2052. 2054. 2056. 2058. 2060. 2062. 2064. 2066. 2068. 2070. 2072. 2074. 2076. 2078. 2080. 2082. 2084. 2086. 2088. 2090. 2092. 2094. 2096. 2098. 2100. 2102. 2104. 2106. 2108. 2110. 2112. 2114. 2116. 2118. 2120. 2122. 2124. 2126. 2128. 2130. 2132. 2134. 2136. 2138. 2140. 2142. 2144. 2146. 2148. 2150. 2152. 2154. 2156. 2158. 2160. 2162. 2164. 2166. 2168. 2170. 2172. 2174. 2176. 2178. 2180. 2182. 2184. 2186. 2188. 2190. 2192. 2194. 2196. 2198. 2200. 2202. 2204. 2206. 2208. 2210. 2212. 2214. 2216. 2218. 2220. 2222. 2224. 2226. 2228. 2230. 2232. 2234. 2236. 2238. 2240. 2242. 2244. 2246. 2248. 2250. 2252. 2254. 2256. 2258. 2260. 2262. 2264. 2266. 2268. 2270. 2272. 2274. 2276. 2278. 2280. 2282. 2284. 2286. 2288. 2290. 2292. 2294. 2296. 2298. 2300. 2302. 2304. 2306. 2308. 2310. 2312. 2314. 2316. 2318. 2320. 2322. 2324. 2326. 2328. 2330. 2332. 2334. 2336. 2338. 2340. 2342. 2344. 2346. 2348. 2350. 2352. 2354. 2356. 2358. 2360. 2362. 2364. 2366. 2368. 2370. 2372. 2374. 2376. 2378. 2380. 2382. 2384. 2386. 2388. 2390. 2392. 2394. 2396. 2398. 2400. 2402. 2404. 2406. 2408. 2410. 2412. 2414. 2416. 2418. 2420. 2422. 2424. 2426. 2428. 2430. 2432. 2434. 2436. 2438. 2440. 2442. 2444. 2446. 2448. 2450. 2452. 2454. 2456. 2458. 2460. 2462. 2464. 2466. 2468. 2470. 2472. 2474. 2476. 2478. 2480. 2482. 2484. 2486. 2488. 2490. 2492. 2494. 2496. 2498. 2500. 2502. 2504. 2506. 2508. 2510. 2512. 2514. 2516. 2518. 2520. 2522. 2524. 2526. 2528. 2530. 2532. 2534. 2536. 2538. 2540. 2542. 2544. 2546. 2548. 2550. 2552. 2554. 2556. 2558. 2560. 2562. 2564. 2566. 2568. 2570. 2572. 2574. 2576. 2578. 2580. 2582. 2584. 2586. 2588. 2590. 2592. 2594. 2596. 2598. 2600. 2602. 2604. 2606. 2608. 2610. 2612. 2614. 2616. 2618. 2620. 2622. 2624. 2626. 2628. 2630. 2632. 2634. 2636. 2638. 2640. 2642. 2644. 2646. 2648. 2650. 2652. 2654. 2656. 2658. 2660. 2662. 2664. 2666. 2668. 2670. 2672. 2674. 2676. 2678. 2680. 2682. 2684. 2686. 2688. 2690. 2692. 2694. 2696. 2698. 2700. 2702. 2704. 2706. 2708. 2710. 2712. 2714. 2716. 2718. 2720. 2722. 2724. 2726. 2728. 2730. 2732. 2734. 2736. 2738. 2740. 2742. 2744. 2746. 2748. 2750. 2752. 2754. 2756. 2758. 2760. 2762. 2764. 2766. 2768. 2770. 2772. 2774. 2776. 2778. 2780. 2782. 2784. 2786. 2788. 2790. 2792. 2794. 2796. 2798. 2800. 2802. 2804. 2806. 2808. 2810. 2812. 2814. 2816. 2818. 2820. 2822. 2824. 2826. 2828. 2830. 2832. 2834. 2836. 2838. 2840. 2842. 2844. 2846. 2848. 2850. 2852. 2854. 2856. 2858. 2860. 2862. 2864. 2866. 2868. 2870. 2872. 2874. 2876. 2878. 2880. 2882. 2884. 2886. 2888. 2890. 2892. 2894. 2896. 2898. 2900. 2902. 2904. 2906. 2908. 2910. 2912. 2914. 2916. 2918. 2920. 2922. 2924. 2926. 2928. 2930. 2932. 2934. 2936. 2938. 2940. 2942. 2944. 2946. 2948. 2950. 2952. 2954. 2956. 2958. 2960. 2962. 2964. 2966. 2968. 2970. 2972. 2974. 2976. 2978. 2980. 2982. 2984. 2986. 2988. 2990. 2992. 2994. 2996. 2998. 3000. 3002. 3004. 3006. 3008. 3010. 3012. 3014. 3016. 3018. 3020. 3022. 3024. 3026. 3028. 3030. 3032. 3034. 3036. 3038. 3040. 3042. 3044. 3046. 3048. 3050. 3052. 3054. 3056. 3058. 3060. 3062. 3064. 3066. 3068. 3070. 3072. 3074. 3076. 3078. 3080. 3082. 3084. 3086. 3088. 3090. 3092. 3094. 3096. 3098. 3100. 3102. 3104. 3106. 3108. 3110. 3112. 3114. 3116. 3118. 3120. 3122. 3124. 3126. 3128. 3130. 3132. 3134. 3136. 3138. 3140. 3142. 3144. 3146. 3148. 3150. 3152. 3154. 3156. 3158. 3160. 3162. 3164. 3166. 3168. 3170. 3172. 3174. 3176. 3178. 3180. 3182. 3184. 3186. 3188. 3190. 3192. 3194. 3196. 3198. 3200. 3202. 3204. 3206. 3208. 3210. 3212. 3214. 3216. 3218. 3220. 3222. 3224. 3226. 3228. 3230. 3232. 3234. 3236. 3238. 3240. 3242. 3244. 3246. 3248. 3250. 3252. 3254. 3256. 3258. 3260. 3262. 3264. 3266. 3268. 3270. 3272. 3274. 3276. 3278. 3280. 3282. 3284. 3286. 3288. 3290. 3292. 3294. 3296. 3298. 3300. 3302. 3304. 3306. 3308. 3310. 3312. 3314. 3316. 3318. 3320. 3322. 3324. 3326. 3328. 3330. 3332. 3334. 3336. 3338. 3340. 3342. 3344. 3346. 3348. 3350. 3352. 3354. 3356. 3358. 3360.

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958 • 2006年4月 第2期 中国农村金融研究 王宇明

— *Wird die Kommission die Möglichkeit haben, die Mitgliedstaaten zu verpflichten, die*

(.11-111) 7. (1) 191 years 11 months 11 days

(3) 若 $\lim_{n \rightarrow \infty} \frac{1}{n} \sum_{k=1}^n f(x_k) = \int_a^b f(x) dx$, 则 $\lim_{n \rightarrow \infty} \frac{1}{n} \sum_{k=1}^n f(x_k) = \int_a^b f(x) dx$.

LONGITUDINAL BEAM AT OUTSIDE OF WALK.

A rectangular cross-section will be used.

Span of beam = 15'-0".

Dead load:

Assume weight of beam = 300# per lineal foot,
and weight of hand rail = 650#/ft.

Weight of sidewalk slab per sq. ft. = 73#.

Weight of slab taken by beam (1/2 sidewalk)-

$$5 \times 15 \times 73 = 5480\#$$

Weight of beam =

$$15 \times 300 = 4500$$

Weight of rail =

$$15 \times 650 = 9750$$

$$\begin{array}{rcl} \text{Live load at } 60\#/ft. & = & 60 \times 5 \times 15 \\ \text{Total} & & \underline{\quad \quad \quad} \\ & & 24230\# \end{array}$$

Max. bending moment = 1/8wl.

$$= 1/8 \times 24200 \times 15 \times 12 = 5453000 \text{ in. lbs.}$$

$$M_s = f_s A x 7/8d.$$

$$M_c = f_c x 1/6bd.$$

Assume "b" = 12"

$$d = \frac{6Mc}{12 \times 600} = \frac{540000 \times 6}{12 \times 600} = 450. \therefore d \approx 1.2$$

use 22".

$$A = \frac{8}{7} = \frac{M_s}{f_s d} = \frac{8 \times 540000}{7 \times 22 \times 16000} = 1.76 \text{ sq. in.}$$

This area is furnished by 5/8" bars spaced 2
c. to c. Total depth of beam should be made 25".

1. Introduction 2. Methodology 3. Results 4. Discussion 5. Conclusion 6. References 7. Appendix 8. Figures 9. Tables 10. Glossary 11. Index 12. Bibliography 13. Acknowledgments 14. Author's Note 15. Contact Information 16. Declaration of Interest 17. Funding Source 18. Data Availability 19. Ethics Statement 20. Conflicts of Interest 21. Author Contributions 22. Correspondence 23. Additional Information 24. Supplementary Materials 25. References

The purpose of this study is to investigate the effects of the proposed intervention on the target population. The study was designed as a randomized controlled trial.

The study was conducted in a controlled environment. The participants were recruited from a local community.

The study was approved by the local ethics committee. The participants provided informed consent.

The study was conducted over a period of 12 weeks. The participants were assigned to two groups: the intervention group and the control group.

The intervention group received the proposed intervention. The control group received a placebo.

The primary outcome measure was the change in the target variable. The secondary outcome measures were the change in the other variables.

The data were analyzed using statistical software. The results are presented in the following sections.

The results of the study are presented in the following sections. The first section presents the results of the primary outcome measure.

The results of the study are presented in the following sections. The second section presents the results of the secondary outcome measures.

The results of the study are presented in the following sections. The third section presents the results of the other variables.

The results of the study are presented in the following sections. The fourth section presents the results of the other variables.

The results of the study are presented in the following sections. The fifth section presents the results of the other variables.

The results of the study are presented in the following sections. The sixth section presents the results of the other variables.

The results of the study are presented in the following sections. The seventh section presents the results of the other variables.

The results of the study are presented in the following sections. The eighth section presents the results of the other variables.

The results of the study are presented in the following sections. The ninth section presents the results of the other variables.

The results of the study are presented in the following sections. The tenth section presents the results of the other variables.

The results of the study are presented in the following sections. The eleventh section presents the results of the other variables.

The results of the study are presented in the following sections. The twelfth section presents the results of the other variables.

-DESIGN OF CANTILEVER-

This will be designed as a cantilever beam having a single load at the end, due to the weight of the beam just designed.

Weight of one girder, including slab and rail 24000#

The general dimension of the beam will be assumed, and the reinforcing figured ; as it will be of such shape as to be of nearly uniform strength, the uniform load due to its own weight will not be considered.

Maximum moment due to load P at end M Pl.

$M = 24000 \times 15 = 360000 \text{ ft. lbs.} = 4320000 \text{ in. lbs.}$

Shear at any point = 24000#. Therefore the required area at an allowable shearing stress of 100#/sq.in. = 240 sq.in. Make the beam 12x20 at least.

$$A = \frac{8}{7} \frac{M_s}{F_s d}$$

-FLOOR SLAB-

The live load upon the floor slab is 100#/sq.ft. as per specifications. From Table 21, Turneaure and Maurer, as for the sidewalk slab, the thickness

-Page 1 of 1-

This will be the first of a series of

reports on the progress of the work.

The first report will be on the

work done during the first year.

The second report will be on the

work done during the second year.

The third report will be on the

work done during the third year.

The fourth report will be on the

work done during the fourth year.

The fifth report will be on the

work done during the fifth year.

The sixth report will be on the

work done during the sixth year.

The seventh report will be on the

$$I = \frac{C}{T}$$

-Page 2 of 2-

This will be the second of a series of

reports on the progress of the work.

The first report will be on the

of slab for a span of 11.5 ft. and a loading of 200#/ft. ($M \frac{1}{12} w l^2$) is 8". .547 sq. inches of steel are required ; this is furnished by $\frac{3}{8}$ " bars, 2" apart.

-GIRDER-

This will be designed as a T beam with a flange thickness of 8" (floor slab.)

Assume weight of girder = 1000# per lineal ft.

Weight of slab 98 " " "

Live load 100# " " "
(consider as dead load) 198#

Dead load on girder

Slab at 198#/sq. ft. 1980#

Girder

Total- $\frac{1000}{2980\#}$

Bending moment $M_D = \frac{1}{8} \times 2980 \times 15^2 \times 12 = 1008000\#'$

The live load is furnished by a sprinkler weighing 42 Tons per car. This is on two trucks 16'4" c. to c. 21 Tons per truck. The maximum moment occurs with the load at the centre.

[illegible]

Figure 1

(continued)

[illegible]

$$M_L = 42000 \times 15/4 \times 12 = 1890000 \text{ #}$$

$$\begin{aligned} \text{Total moment} &= M_D + M_L = 1890000 + 1008000 \\ &= 2898000 \text{ #} \end{aligned}$$

-SHEAR-

The maximum shear occurs at the supports when the truck is just leaving the span.

$$\text{This shear is 21 tons} = 42000 \text{ #}$$

$$\text{Dead load shear} = 2980 \times 15/2 = 21400$$

$$\frac{63400 \text{ #}}{\text{Total shear}}$$

$$\text{Allowable shearing stress} = 100 \text{ #/sq.in.}$$

$$\text{Area of concrete required} = 634 \text{ sq. in.}$$

Owing to the arched form of the spandrels, this area will be provided for at the ends. It will never be required at the center, as the sketch Fig. "C", Pl. 11. shows. The maximum shear which can exist at the center of the span is $42000/2 = 21000 \text{ #}$, and it may be considered as varying uniformly towards the end. This shear would require only 210 sq. in. We will use 560^{sq.} and assume that the arch will take the shear between the quarter point and the supports.

$$560 \text{ sq. in.} = 14" \times 40", 16" \times 35", \text{ or } 20" \times 28" \text{ section}$$

Try 16" x 35", with 8" flange. From plate X, Page 284, Tureaure and Maurer, for $t/d = 8/35 = .228$, a

1914-1915

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1916-1917

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1917-1918

1918-1919

1919-1920

1920-1921

1921-1922

1922-1923

1923-1924

1924-1925

1925-1926

1926-1927

1927-1928

1928-1929

1929-1930

1930-1931

1931-1932

1932-1933

1933-1934

1934-1935

and $f_c=600$, $j=.905$ and $M/bd^2=85$

$$jd = .905 \times 35 = 31.7$$

$$bd^2 = 2898000/85 = 34100 \therefore b = 34100/35 = 27.9" \text{ say } 28"$$

$$A = \frac{M}{f_j d} = \frac{2898000}{31.7 \times 16000} = 5.71 "$$

Use 6 at 7/8" 3.60 and 5 at 3/8" 2.20, Total

area 5.80^{sq} See figure "D" Pl. II for arrangement of rods. Distance of centre of gravity of reinforcement from \bar{b} of bottom row = $2.2 \times 2.5 / 5.80 = 1.53$

Total depth of girder $35 + 1.53 + 2.5 = 39"$.

These rods will be turned up at intervals, to assist in overcoming the shear. The points at which they may be turned up are found from the following formula:

$$X = 1/A \sqrt{a_1 + a_2 + a_3 + \dots + a_n}$$

X being the unbent lengths of rod required to resist bending moment and a_1, a_2 , etc. the areas of the rods.

-LENGTHS-

| No. of rod. | $a_1 + a_2 + \dots + a_n$ | X
4.13 ft. |
|-------------|---------------------------|---------------|
| 1 | .44 | |
| 2 | .88 | 5.85 " |
| 3 | 1.32 | 7.15 " |
| 4 | 1.76 | 8.26 " |
| 5 | 2.20 | 9.25 " |
| 6 | 2.80 | 10.42 " |
| 7 | 3.40 | 11.50 " |
| 8 | 4.00 | 12.45 " |
| 9 | 4.60 | 13.35 " |
| 10 | 5.20 | 14.20 " |
| 11 | 5.80 | 15.00 " |

The length of rod required to develop a bond strength equal to the working strength equals $16000/4 \times 75 = 53.4d$. This 47" for a 7/8" rod; and it is well provided for, as shown by the above table. The rods will be arranged as follows, the unbent lengths being given in each case:

| | | |
|---|----------|---|
| 2 | at 6'-0" | |
| 2 | " 9'-0" | |
| 2 | " 11'-0" | |
| 2 | " 13'-0" | } These rods to continue over supports. |
| 3 | " 15'-0" | |

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We will now find the point at which the shearing stress becomes equal to $100\%/sq.in.$ as beyond that point stirrups will be required. The stress at the centre equals $42000 / 560 = 75\%/per sq.in.$

The maximum shear at any point distant x from the centre $= 3X / 15 \times 21400 + 3X / 15 \times 42000 + 42000$, and it becomes equal to $100\%/sq.in.$ when $56000 - 42000 = 2X / 15 \times 63400$.

$$3X / 15 \times 63400 = 14000 \therefore x = 1.65$$

Stress carried by concrete equals $30\%/sq.in.$

" " " steel " 70% " "

Use $1/2"$ stirrups in double loop. Value at $12000\%/sq.in. = 12000 \times 4 \times .2 = 9600\%$ Spacing $= 12000 / 70 \times 1.6 = 3.6$

Space $9"$ apart throughout.

-INVESTIGATION OF FLANGE FOR SHEAR-

The width of the web of the girder is less than the width of track c.toe. of rails so that the floor slab under the wheel is the condition of a beam with a load P at the point where the rail lies. It will be considered for purposes of investigation as a cantilever with a load P at the end.

Its length equals $(53.5-16)\frac{1}{2} \times 1.6 = 30.35"$. The weight of one wheel is $13000/4 = 10500\text{ lbs}$.

Bending moment equals $10500 \times 30.35 = 313500\text{ in-lbs}$.
 Shear = 10500 lbs . Area required equals $10500 / 105/3 = 13\frac{1}{3}"$. This is the required width of slab over which a single wheel load must be distributed if the 8" slab is to be safe. As the wheels are more than 14" apart., the 8" slab assumed should be safe enough. Add 45 fillet on each side of girder.

Area of steel required.

$$A = \frac{M}{f_s d} = \frac{313500}{77 \times 51.5/3} = 1.77\text{ in}^2 \quad \text{Use } 5/8\text{ rods}$$

3" apart.

See Fig.F-P1. 11.

-ABUTMENT-

(See Plate 4.)

Figure "A".

In figuring size of abutment it is necessary to know the maximum thrust at springing line due to arch action and also the vertical force acting downwards of that amount of concrete which we will consider as consisting the abutment.

Column marked "C" and block of solid concrete marked "B" in the sketch are those portions which shall constitute the abutment. (Column C is hollow as shown in both figs. A and B, its interior being filled completely with earth A.)

-VERTICAL FORCE-

Weight of column marked "C".

Earth: $44' \times 9' \times 37' \times 120 \frac{\#}{\text{cu ft}} = 1758240 \frac{\#}{\text{cu ft}}.$

Concrete: $(2 \times 3' \times 12' \times 44' \times 150 \frac{\#}{\text{cu ft}})$
 $(2 \times 3' \times 37' \times 44' \times 150 \frac{\#}{\text{cu ft}}) = 1549800 \frac{\#}{\text{cu ft}}$

Total weight $= 1758240 \frac{\#}{\text{cu ft}} + 1549800 \frac{\#}{\text{cu ft}} = 3308000 \frac{\#}{\text{cu ft}}.$

Weight per foot width of bridge:

$3308000 \div 33 = 100000 \frac{\#}{\text{ft}} = 100 \text{ kips'}$

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(see page 2)

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Weight of block "B":

$$30' \times 32' \times 1' \times 150 \frac{\text{#}}{\text{ft}^3} = 144000 \frac{\text{#}}{\text{ft. width of bridge.}}$$

Total downward vertical pressure:

$$144000 \frac{\text{#}}{\text{ft}} + 100000 \frac{\text{#}}{\text{ft}} = 244 \text{ kips}$$

Center of gravity of the two portions, column "C" and block "B" is found and from it is drawn vertically downward a line. Maximum axial thrust of arch (217 k.) is drawn in direction of action. This line cuts the vertical line at point as shown in Fig. A. From this point, and to a certain scale is laid off 244 K. & 217 K.

Their resultant as shown must and does pass within the middle third of the block "B". Therefore the dimension of "B" and "C" are correct.

-PILES-

Vertical downward force of abutment as found above : 3,308,000# (See Fig. "C" Plate 4)

Formula for resistance "R" of one pile:

$$R = \frac{2wh}{s+1} \quad h = 20' = \text{drop of hammer.} \quad W = 3000 \frac{\text{#}}{\text{weight of hammer.}} \quad s = 1' = \text{distance pile is imbedded}$$

at last blow of hammer.

$$R = \frac{2 \times 3046 \times 20}{1+1} = 60920 \frac{\text{#}}{\text{pile}} \quad \frac{3308000}{60920} = 56 \text{ piles}$$

necessary.

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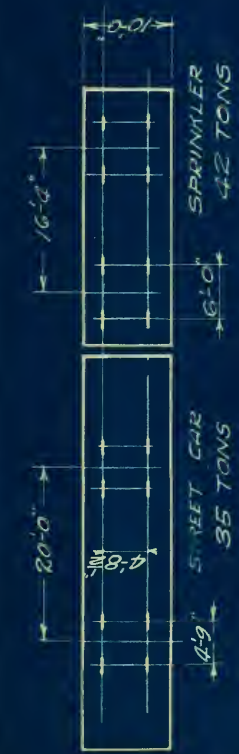


FIGURE "A"



FIGURE "C"

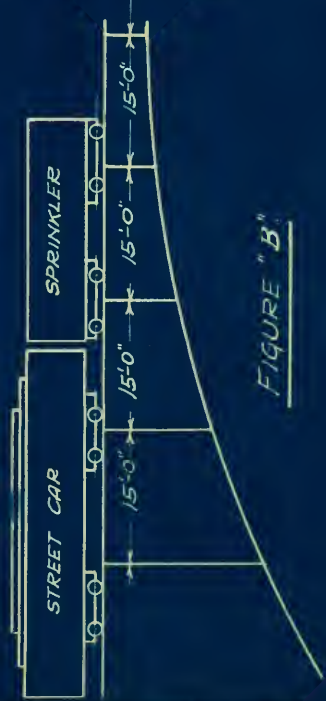


FIGURE "B"

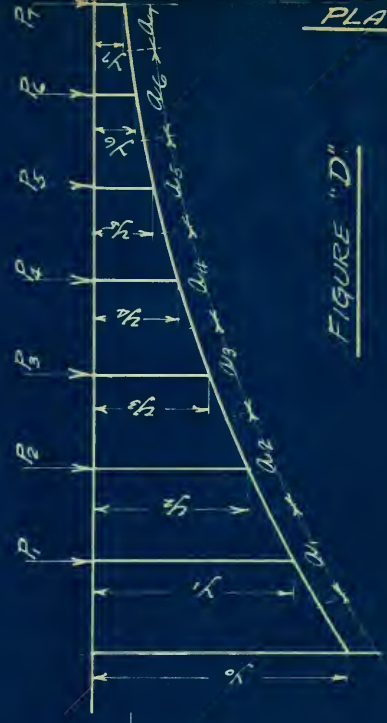


FIGURE "D"



FIGURE "A"
SIDEWALK BRACKET



FIGURE "B"
SIDE WALK

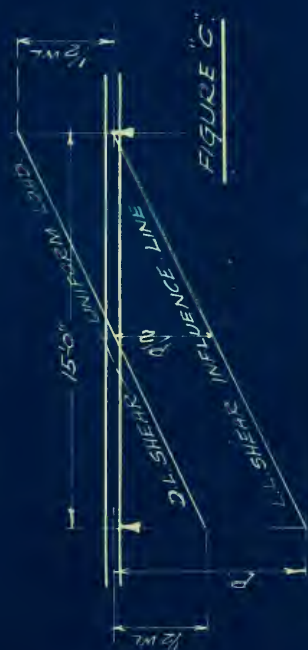


FIGURE "C"

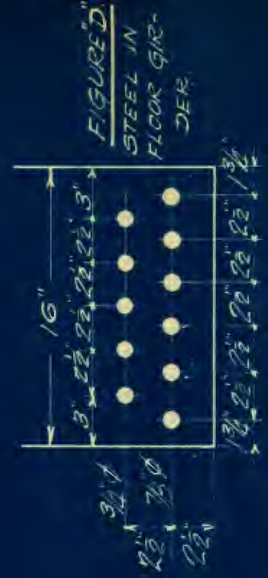


FIGURE "D"
STEEL IN
FLOOR GIRDER



FIGURE "E"

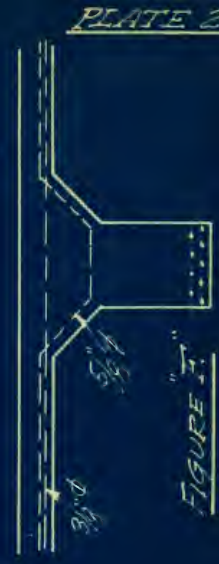
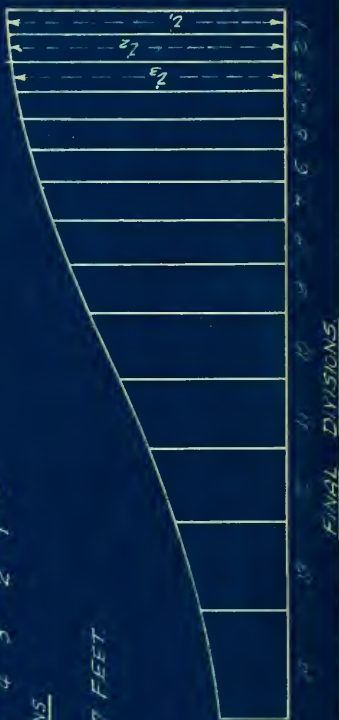
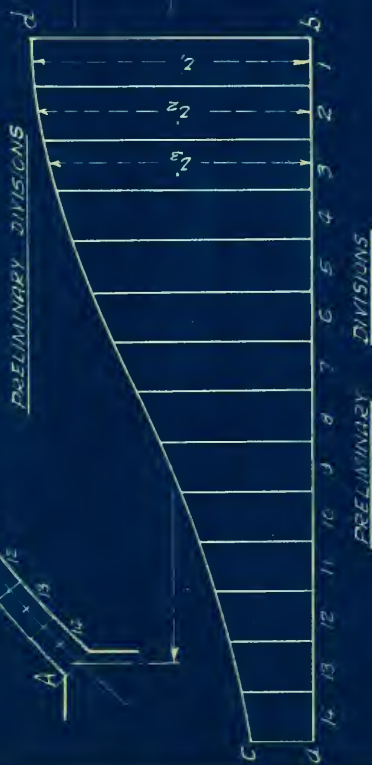


FIGURE "F"



LENGTH ARC AB = 20 = 1472.6 = 116.97 FEET

$$\sum_{N=1}^{14} \frac{1}{N} = 2.074$$

$$20 \cdot 2 = 2.074$$

SEE TABLE A

Fig. B

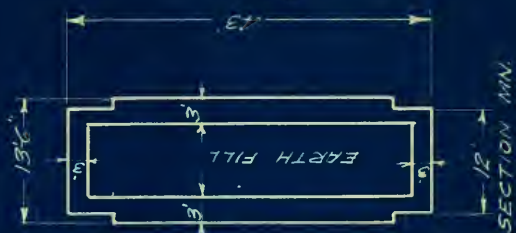
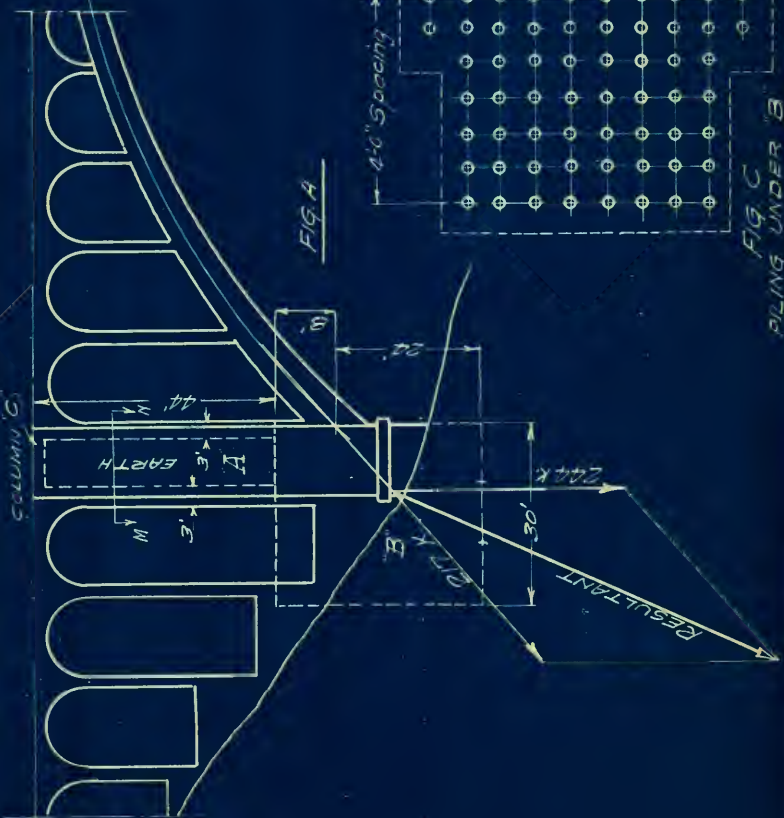


FIG. A





PROPERTIES OF PRELIMINARY EQUAL DIVISIONS

| PROPERTIES OF PRELIMINARY EQUAL DIVISIONS | | | | PROPERTIES OF FINAL DIVISIONS | | | | | |
|---|---------|--------|---------|-------------------------------|---------------------|-------|---------|--------|------|
| NO OF DIVIS | DEPTH D | I_c | $15I_s$ | $I = I_c + 15I_s$ | $i = \frac{I}{I_c}$ | i | d_s | I | d |
| 1 | 3.00 | 2.251 | .253 | 2.504 | .993 | .408 | 4.162 | 2.445 | 3.01 |
| 2 | 3.01 | 2.272 | .253 | 2.530 | .962 | .4032 | 4.200 | 2.48 | 3.02 |
| 3 | 3.09 | 2.457 | .264 | 2.723 | .9667 | .394 | 4.526 | 2.515 | 3.04 |
| 4 | 3.18 | 2.676 | .285 | 2.961 | .9376 | .380 | 4.821 | 2.562 | 3.06 |
| 5 | 3.23 | 2.809 | .296 | 3.104 | .9222 | .3801 | 5.051 | 2.631 | 3.09 |
| 6 | 3.28 | 2.944 | .304 | 3.248 | .9075 | .3680 | 5.347 | 2.712 | 3.12 |
| 7 | 3.46 | 3.451 | .335 | 3.789 | .8644 | .351 | 6.273 | 2.945 | 3.15 |
| 8 | 3.66 | 4.079 | .373 | 4.457 | .8225 | .328 | 7.382 | 3.05 | 3.20 |
| 9 | 3.78 | 4.508 | .403 | 4.911 | .8035 | .298 | 8.200 | 3.36 | 3.38 |
| 10 | 3.98 | 5.253 | .445 | 5.703 | .7853 | .261 | 9.474 | 3.92 | 3.50 |
| 11 | 4.22 | 6.261 | .495 | 6.756 | .7480 | .218 | 11.352 | 4.58 | 3.62 |
| 12 | 4.41 | 7.139 | .544 | 7.683 | .7309 | .177 | 12.732 | 5.65 | 3.90 |
| 13 | 4.69 | 8.538 | .616 | 9.154 | .7093 | .135 | 15.25 | 7.410 | 4.28 |
| 14 | 4.96 | 10.172 | .695 | 10.867 | .6922 | .0981 | 18.050 | 10.200 | 4.75 |
| | | | | | 3.4769 | | 116.974 | | |

5% STEEL ABOVE & BELOW AXIS

AVERAGE

LENGTH

ARC

K_P =

116.97

$$i = \frac{\sum i}{n} = \frac{3.4769}{12} = .2897$$

$$d_s i = 116.97 \times .2897 = 33.84$$

$$d_s i = 116.97 \times .2897 = 33.84$$

(SEE PLATE 3)

TABLE "B"

| POINT | x | y | x^2 | xy | y^2 | M_L | M_R | $[M_L + M_R] y$ | $[M_L - M_R] x$ |
|----------|--------|---------|----------|------|----------|------------|----------|-----------------|-----------------|
| 1 | 2.281 | .0011 | 4.331 | | .0000 | - 44.52 | - 44.52 | - .069 | 2.0 |
| 2 | 8.500 | 0.202 | 39.199 | | .0408 | - 96.01 | - 96.01 | - 38.784 | 0 |
| 3 | 13.200 | .411 | 112.876 | | .169 | - 165.00 | - 165.00 | - 131.520 | 0 |
| 4 | 14.50 | .906 | 235.56 | | .821 | - 243.3 | - 243.3 | - 420.86 | 0 |
| 5 | 20.584 | 1.521 | 415.52 | | 2.3104 | - 396.2 | - 396.2 | - 1204.448 | 0 |
| 6 | 25.840 | 2.340 | 661.50 | | 5.4756 | - 560.7 | - 560.7 | - 2624.076 | 0 |
| 7 | 31.590 | 3.502 | 991.96 | | 12.250 | - 764.2 | - 764.2 | - 5349.400 | 0 |
| 8 | 38.240 | 5.100 | 1466.49 | | 26.0100 | - 1082.2 | - 1082.2 | - 11038.440 | 0 |
| 9 | 45.616 | 7.262 | 2126.68 | | 52.707 | - 1446.4 | - 1446.4 | - 21088.796 | 0 |
| 10 | 54.150 | 10.260 | 3019.48 | | 105.27 | - 1999.3 | - 1999.3 | - 41025.640 | 0 |
| 11 | 63.6 | 14.310 | 4266.18 | | 204.49 | - 2699.8 | - 2699.8 | - 77274.0 | 0 |
| 12 | 72.92 | 19.72 | 5976.59 | | 388.89 | - 3600.5 | - 3600.5 | - 141984.0 | 0 |
| 13 | 84.9 | 26.396 | 8335.48 | | 696.74 | - 4908.5 | - 4908.5 | - 269168.800 | 0 |
| 14 | 98.34 | 37.680 | 11653.0 | | 1418.680 | - 6538.6 | - 6538.6 | - 493002.9 | 0 |
| Σ | | 129.611 | 39324.83 | | 2913.858 | - 49070.46 | | 7054471.733 | 0.0 |

CASE I

SPAN COVERED ENTIRELY WITH LIVE LOAD

$$H_0 = 174.5 \text{ KIPS}$$

$$V_0 = 0$$

$$M_0 = 152 \text{ KIP-Feet}$$

| POINT | THRUST | | ECCENTRIC DIST | | BENDING MOMENT | |
|-------|--------|-------------|----------------|------------|----------------|---------|
| | LEFT | RIGHT | LEFT | RIGHT | LEFT | RIGHT |
| 1 | 174.8 | 174.8 | + .558 | + .558 | + 87.65 | + 87.65 |
| 2 | 174.8 | 174.8 | + .407 | + .407 | + 71.21 | + 71.21 |
| 3 | 174.8 | 174.8 | + .209 | + .209 | + 36.68 | + 36.68 |
| 4 | 176.1 | 176.1 | + .267 | + .267 | + 74.0 | + 74.0 |
| 5 | 176.1 | 176.1 | + .074 | + .074 | + 1.3 | + 1.3 |
| 6 | 176.1 | 176.1 | - .1165 | - .1165 | - 20.5 | - 20.5 |
| 7 | 179.0 | 179.0 | - .113 | - .113 | - 20.2 | - 20.2 |
| 8 | 179.0 | 179.0 | - .335 | - .335 | - 60.0 | - 60.0 |
| 9 | 183.8 | 183.8 | - .252 | - .252 | - 46.4 | - 46.4 |
| 10 | 183.8 | 183.8 | - .373 | - .373 | - 70.3 | - 70.3 |
| 11 | 191.4 | 191.4 | - .350 | - .35 | - 67.8 | - 67.8 |
| 12 | 191.4 | 191.4 | - .149 | - .149 | - 28.5 | - 28.5 |
| 13 | 202.8 | 202.8 | - .817 | - .817 | - 165.5 | - 165.5 |
| 14 | 217.7 | 217.7 | + .788 | + .788 | + 171.4 | + 171.4 |
| | ABOVE | GRAPHICALLY | SHOWN | ON PLATE 5 | | |

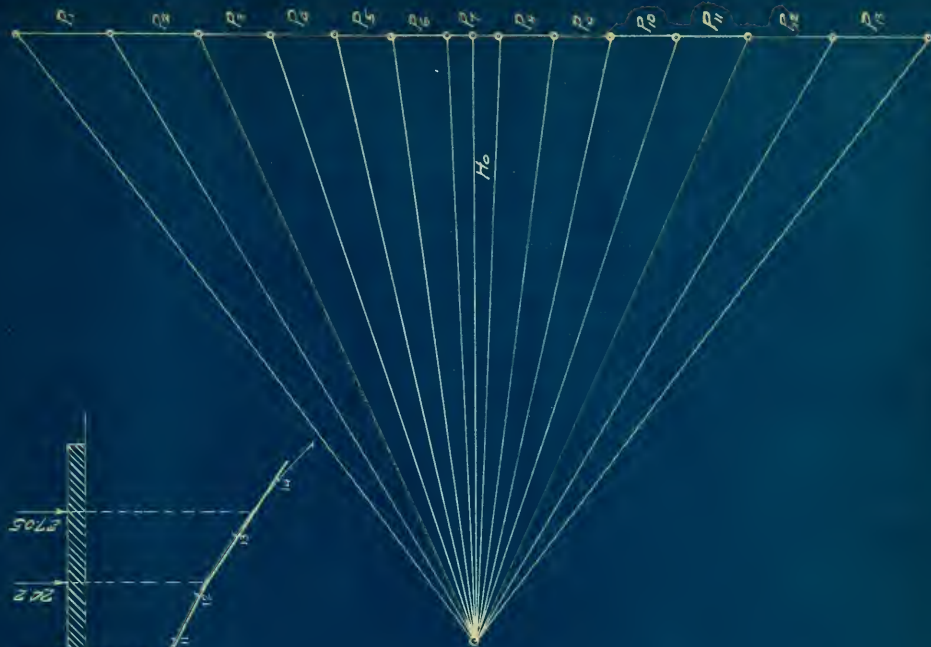
CASE I

SPAN COVERED ENTIRELY WITH LINE LOAD

| POINT | e/h | $M/6h^2 f_c$ | f_c | $1/k$ | f_s |
|----------------------------------|-------|--------------|-------|-------|-------|
| 1 | .185 | .107 | 597 | .98 | 2210 |
| 2 | .134 | .089 | 598 | .84 | 1851 |
| 3 | .0686 | .057 | 532 | .504 | 1632 |
| 4 | .0872 | .068 | 513 | .610 | 1584 |
| 5 | .0242 | .0234 | 403 | .240 | 2460 |
| 6 | .0374 | .0342 | 427 | .320 | 4840 |
| 7 | .0359 | .0339 | 418 | .318 | 5020 |
| 8 | .1048 | .0773 | 522 | .700 | 3718 |
| 9 | .0746 | .0625 | 464 | .560 | 4050 |
| 10 | .1065 | .0778 | 508 | .710 | 4925 |
| 11 | .0974 | .0741 | 410 | .682 | 4120 |
| 12 | .0382 | .0354 | 372 | .320 | 4238 |
| 13 | .1908 | .1089 | 575 | 1.001 | 2184 |
| 14. | .1644 | .1012 | 519 | .941 | 3830. |
| FROM TURN & MARGIN
PLATE XXIV | | | | | |

$$f_s = n f_c \left[1 - \frac{d}{h} \right]$$

CASE I

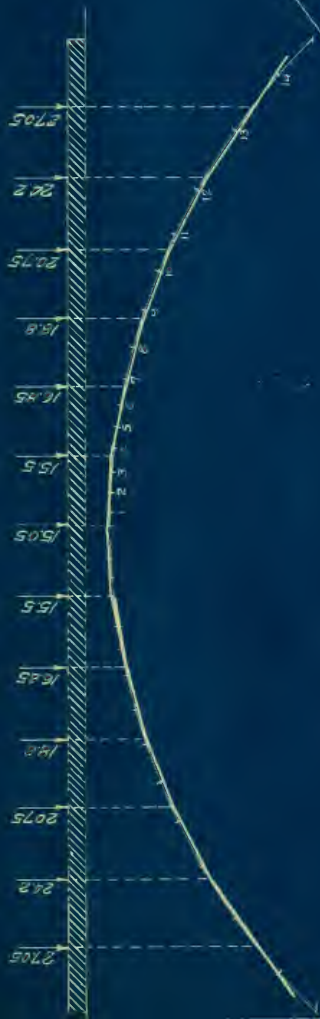


CASE I
LIVE LOAD OVER ENTIRE SPAN

$$H_0 = 174.5 \text{ KIPS}$$

$$V_0 = 0.0$$

$$M_0 = 132 \text{ KIP-FEET}$$



| PT | X | Y | X ² | Y ² | M _X | M _Y | [M _X +M _Y] | [M _X -M _Y] |
|----|--------|---------|----------------|----------------|----------------|----------------|-----------------------------------|-----------------------------------|
| 1 | 2.281 | .001 | 4.331 | .0000 | - 34.52 | - | 0.69 | 0.0 |
| 2 | 8.5 | .201 | 39.199 | .0408 | - 96.010 | - | 38.784 | 0. |
| 3 | 13.2 | .211 | 112.87 | .0446 | - 165.00 | - | 131.52 | 0. |
| 4 | 14.5 | .906 | 235.56 | .821 | - 243.3 | - | 440.123 | + 26.82 |
| 5 | 20.584 | 1.521 | 423.52 | 2.310 | - 375.4 | - | 1172.832 | + 220.104 |
| 6 | 25.84 | 2.36 | 667.50 | 5.476 | - 500.7 | - | 2527.551 | + 1076.625 |
| 7 | 31.59 | 3.502 | 997.96 | 12.25 | - 765.2 | - | 4932.90 | + 3728.5 |
| 8 | 38.24 | 5.1 | 1466.49 | 26.01 | - 1082.2 | - | 10445.82 | + 2450.46 |
| 9 | 45.616 | 7.262 | 2126.68 | 52.707 | - 1397.3 | - | 19199.433 | + 6910.0 |
| 10 | 54.15 | 10.26 | 3019.48 | 105.27 | - 1999.3 | - | 38211.318 | + 15042.0 |
| 11 | 63.60 | 14.31 | 4266.18 | 204.49 | - 2699.8 | - | 71485.605 | + 28762.30 |
| 12 | 72.02 | 14.72 | 5976.59 | 308.89 | - 3600.5 | - | 131197.16 | + 39960.16 |
| 13 | 84.90 | 26.306 | 8357.45 | 696.74 | - 4908.5 | - | 237867.2 | + 65380.0 |
| 14 | 98.34 | 37.68 | 11653.0 | 1418.68 | - 6538.6 | - | 353704.88 | + 01062.0 |
| Σ | | 129.611 | 39324.83 | 2013.862 | -24406.13 | -19004.08 | -971355.195 | +264229.029 |
| | | | | | -43490.21 | | | |

CASE II

LIVE LOAD ON LEFT HALF ONLY

$$H_c = 165.5 \text{ KIPS}$$

$$V_c = 336 \text{ "}$$

$$M_c = 207 \text{ KIP-FEET}$$

TABLE "D"

| POINT | THRUST | | ECCENTRIC DIST | | BENDING | | MOMENT |
|-------|------------------------------------|-------|----------------|--------|---------|-------|--------|
| | LEFT | RIGHT | LEFT | RIGHT | LEFT | RIGHT | |
| 1 | 165.6 | 165.4 | -.0362 | -.1288 | -5.99 | | -21.3 |
| 2 | 165.6 | 165.4 | -.279 | -.426 | -46.43 | | -70.5 |
| 3 | 165.6 | 165.4 | -.1928 | -.438 | -31.9 | | -72.4 |
| 4 | 166.7 | 166.6 | -.1437 | -.480 | -23.9 | | -80.2 |
| 5 | 166.7 | 166.6 | -.328 | -.434 | -54.7 | | -72.3 |
| 6 | 166.7 | 166.6 | -.399 | -.528 | -65.7 | | -55.1 |
| 7 | 169.5 | 169.0 | -.338 | -.476 | -57.3 | | -50.6 |
| 8 | 169.5 | 169.0 | -.579 | -.537 | -80.0 | | -90.8 |
| 9 | 174.8 | 172.9 | -.1218 | -.625 | -21.3 | | -108.1 |
| 10 | 174.8 | 172.9 | -.557 | -.621 | -97.6 | | -107.3 |
| 11 | 182.0 | 178.1 | -.521 | -.663 | -95.1 | | -118.5 |
| 12 | 182.0 | 178.1 | -.352 | -.663 | -69.8 | | -11.9 |
| 13 | 194.0 | 186.8 | -.915 | -.176 | -151.0 | | -30.3 |
| 14 | 209.4 | 193.5 | + .176 | +1.100 | +36.9 | | +29.0 |
| | ABOVE SHIPWRIGHTS SHOWN ON PLATE 6 | | | | | | |

CASE II
 LIVE LOAD ON LEFT HALF ONLY

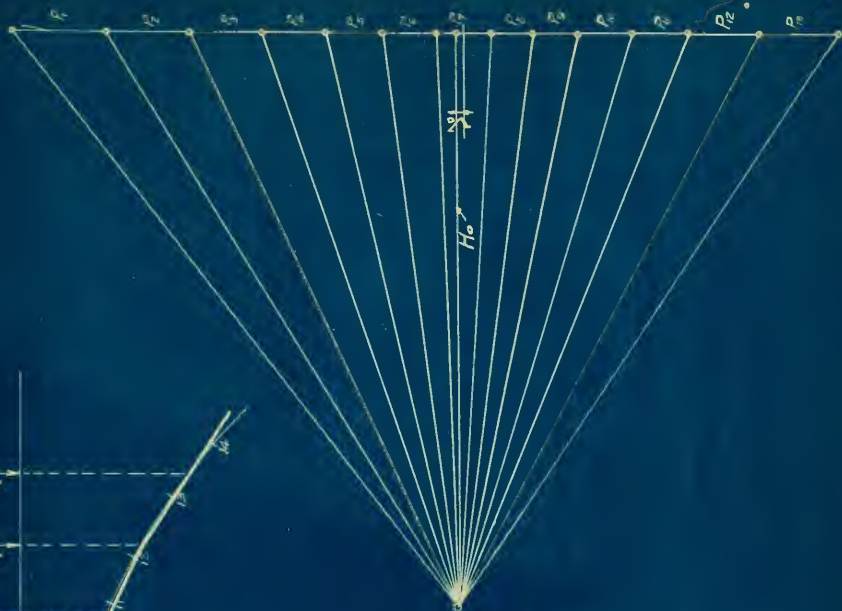
| POINT | e/h | $M/\frac{1}{6}hf_c$ | f_c | $1/k$ | f_s |
|---------------------------------|-------|---------------------|-------|-------|-------|
| 1 | .0121 | .0151 | 303 | .120 | 4110 |
| 2 | .0924 | .0718 | 463 | .642 | 3630 |
| 3 | .0632 | .0557 | 427 | .481 | 3890 |
| 4 | .0470 | .0426 | 416 | .391 | 4112 |
| 5 | .162 | .0995 | 401 | .922 | 2810 |
| 6 | .1294 | .083 | 539 | .818 | 2998 |
| 7 | .1072 | .0778 | 574 | .710 | 3260 |
| 8 | .1623 | .0996 | 600 | .930 | 2740 |
| 9 | .0361 | .0348 | 372 | .311 | 4290 |
| 10 | .158 | .0981 | 561 | .914 | 2480 |
| 11 | .143 | .0927 | 537 | .862 | 3072 |
| 12 | .098 | .0741 | 432 | .780 | 3262 |
| 13 | .213 | .1158 | 592 | 1.210 | 1840 |
| 14 | .037 | .0351 | 327 | .310 | 3772 |
| FROM TABLE 2.1M
PLATE VII-3A | | | | | |

CASE II [LEFT]
 $f_s = n f_c [1 - d/kh]$

| POINT | e_h | M/bh^2f_c | f_c | $1/k$ | f_s |
|---|-------|-------------|-------|-------|-------|
| 1 | .0428 | .0381 | 428 | .362 | 4700 |
| 2 | .1410 | .0921 | 582 | .860 | 3124 |
| 3 | .144 | .093 | 579 | .872 | 3086 |
| 4 | .158 | .0992 | 597 | .895 | 2960 |
| 5 | .141 | .093 | 564 | .860 | 3240 |
| 6 | .170 | .103 | 599 | .950 | 2594 |
| 7 | .151 | .0957 | 588 | .890 | 2896 |
| 8 | .168 | .102 | 567 | .948 | 2640 |
| 9 | .185 | .108 | 602 | .981 | 2408 |
| 10 | .178 | .106 | 571 | .970 | 2562 |
| 11 | .182 | .106 | 509 | .981 | 2432 |
| 12 | .017 | .018 | 295 | .150 | 3915 |
| 13 | .0428 | .0385 | 335 | .358 | 2982 |
| 14 | .232 | .1197 | 559 | 1.210 | 1430 |
| FROM TABLE A AND C
PLATES 200 to 206 | | | | | |

CASE II [RIGHT]
 $f_s = n f_c \left[1 - \frac{d}{kh} \right]$





CASE II
LIVE LOAD ON LEFT HALF ONLY

$$\begin{array}{l} H_0 = 165.5 \\ V_0 = 336 \\ M_0 = 20.7 \end{array}$$



TABLE "F"

| PT | χ | ψ | χ^2 | ψ^2 | m_1 | m_R | $[m_1 + m_R] \psi$ | $[m_R - m_1] \chi$ |
|----------|--------|---------|----------|----------|-----------|----------|--------------------|--------------------|
| 1 | 2.281 | .001 | 4.331 | .0000 | - 44.52 | - 44.52 | - .069 | 0.0 |
| 2 | 8.5 | .201 | 39.194 | .0408 | 96.01 | - 96.01 | - 38.782 | 0.0 |
| 3 | 13.2 | .411 | 112.87 | .169 | 165.0 | - 165.0 | - 131.52 | 0. |
| 4 | 14.5 | .906 | 235.56 | .821 | 243.3 | - 243.3 | - 140.86 | 0. |
| 5 | 20.584 | 1.521 | 415.52 | 2.310 | 346.2 | - 346.2 | - 1204.448 | 0. |
| 6 | 25.84 | 2.34 | 681.50 | 5.4766 | 560.7 | - 560.7 | - 2624.076 | 0. |
| 7 | 31.59 | 3.502 | 991.96 | 12.25 | 764.2 | - 764.2 | - 5349.4 | 0. |
| 8 | 38.24 | 5.10 | 1466.49 | 26.01 | 1082.2 | - 1082.2 | - 11038.44 | 0. |
| 9 | 45.601 | 7.264 | 2126.68 | 52.707 | 1443.6 | - 1443.6 | - 20961.072 | 0. |
| 10 | 54.15 | 10.26 | 3019.48 | 105.27 | 1963.8 | - 1963.8 | - 40249.176 | 0. |
| 11 | 63.60 | 14.31 | 4266.18 | 204.490 | 2615.1 | - 2615.1 | - 74844.162 | 0. |
| 12 | 72.92 | 19.72 | 5976.59 | 308.89 | 3436.0 | - 3436.0 | - 135515.84 | 0. |
| 13 | 84.90 | 26.396 | 8335.48 | 896.74 | 4610.7 | - 4610.7 | - 213444.96 | 0. |
| 14 | 98.34 | 37.68 | 11653.0 | 1418.68 | 6066.9 | - 6066.9 | - 457201.90 | 0. |
| Σ | | 129.611 | 39324.83 | 2913.858 | -46956.46 | | -973092.707 | 0. |

CASE III

LIVE LOAD ON MIDDLE THIRD

$$H_0 = 156.7 \text{ KIPS}$$

$$V_0 = 0$$

$$M_0 = 226.4 \text{ KIP-FEET}$$



| POINT | THRUST | | ECCENTRIC DIST | | BENDING MOMENT | |
|------------------------------------|--------|-------|----------------|---------|----------------|----------|
| | LEFT | RIGHT | LEFT | RIGHT | LEFT | RIGHT |
| 1 | 157.0 | 157.0 | + .572 | + .572 | + 90.10 | + 90.10 |
| 2 | 157.0 | 157.0 | + .483 | + .483 | + 76.204 | + 76.204 |
| 3 | 157.0 | 157.0 | + .426 | + .426 | + 67.8 | + 67.8 |
| 4 | 158.7 | 158.7 | + .6208 | + .6208 | + 98.7 | + 98.7 |
| 5 | 158.7 | 158.7 | + .456 | + .456 | + 72.354 | + 72.354 |
| 6 | 158.7 | 158.7 | + .204 | + .204 | + 32.38 | + 32.38 |
| 7 | 161.8 | 161.8 | + .0661 | + .0661 | + 10.65 | + 10.65 |
| 8 | 161.8 | 161.8 | - .35 | - .35 | - 56.63 | - 56.63 |
| 9 | 166.4 | 166.4 | - .478 | - .478 | - 79.56 | - 79.56 |
| 10 | 166.4 | 166.4 | - .6769 | - .6769 | - 109.7 | - 109.7 |
| 11 | 172.9 | 172.9 | - .74 | - .74 | - 127.9 | - 126 |
| 12 | 172.9 | 172.9 | - .708 | - .708 | - 122.6 | - 122.6 |
| 13 | 182.3 | 182.3 | - .955 | - .955 | - 179.1 | - 179.1 |
| 14 | 194.9 | 194.9 | + .348 | + .348 | + 67.04 | + 67.04 |
| ABOVE GRAPHICALLY SHOWN ON PLATE 7 | | | | | | |

CASE III

LIVE LOAD ON MIDDLE THIRD



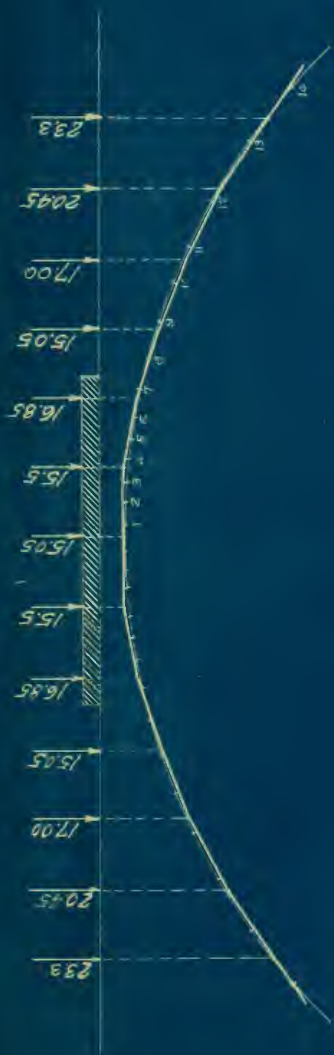
| POINT | $\frac{e}{h}$ | $\frac{M}{bh^2 f_c}$ | f_c | $\frac{1}{K}$ | f_s |
|-----------------------------------|---------------|----------------------|-------|---------------|-------|
| 1 | .941 | .119 | 598 | 1.020 | 2230 |
| 2 | .1634 | .107 | 581 | .930 | 2905 |
| 3 | .140 | .092 | 550 | .860 | 2800 |
| 4 | .203 | .115 | 603 | 1.040 | 3004 |
| 5 | .150 | .095 | 557 | .890 | 2785 |
| 6 | .065 | .056 | 412 | .490 | 4120 |
| 7 | .020 | .022 | 339 | .210 | 4400 |
| 8 | .110 | .078 | 492 | .740 | 3250 |
| 9 | .140 | .092 | 525 | .860 | 2953 |
| 10 | .182 | .105 | 586 | .980 | 2510 |
| 11 | .198 | .112 | 608 | 1.101 | 2250 |
| 12 | .181 | .105 | 533 | .981 | 1804 |
| 13 | .227 | .120 | 563 | 1.10 | 1589 |
| 14 | .071 | .059 | 350 | .551 | 3150. |
| FROM TURN & MOUR
PLATE 20E-20V | | | | | |

CASE III

$$f_s = m f_c \left[1 - \frac{d}{h} \right]$$

$$f_c \text{ from } \frac{M}{bh^2 f_c}$$





CASE III
LIVE LOAD OVER MIDDLE THIRD.

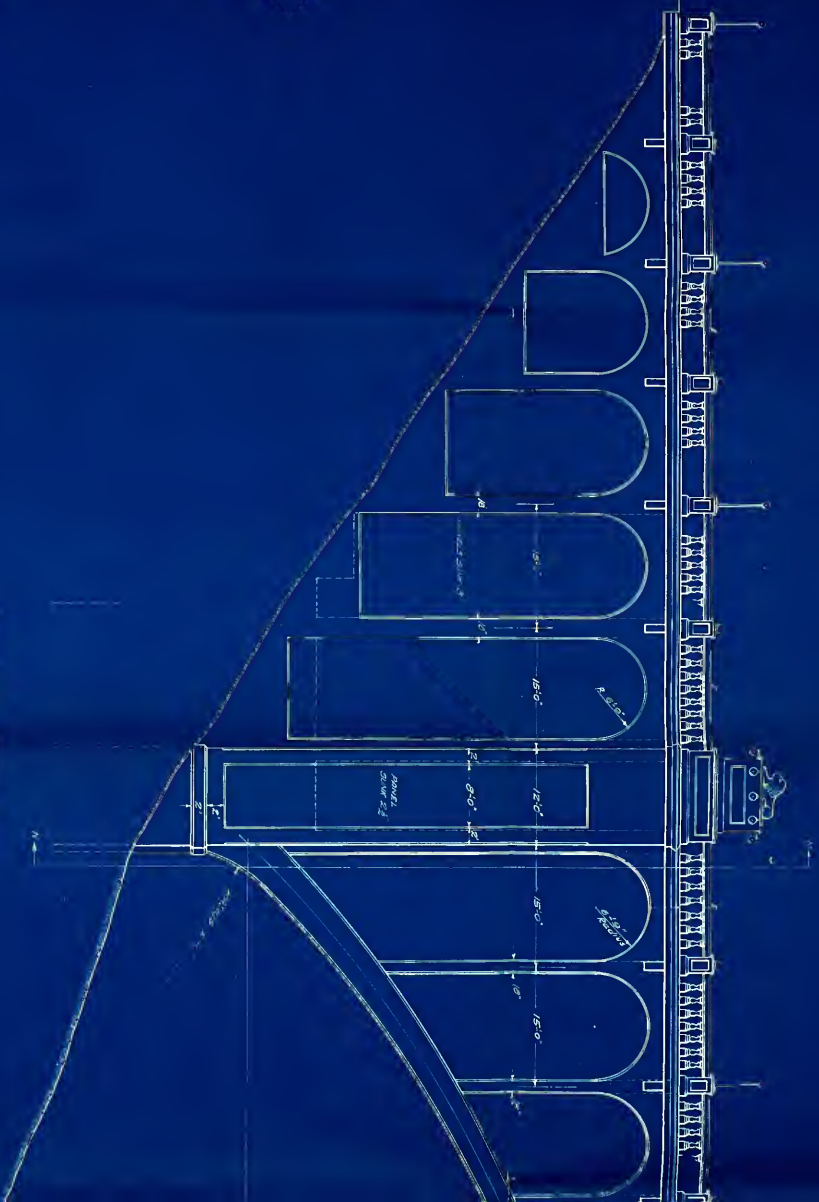
$$\begin{aligned} H_0 &= 155.7 \text{ KIPS} \\ V_0 &= 0.0 \\ M_0 &= +226.4 \text{ KIP-FEET} \end{aligned}$$

H_0

1000



July 1891



ONE-HALF ELEVATION

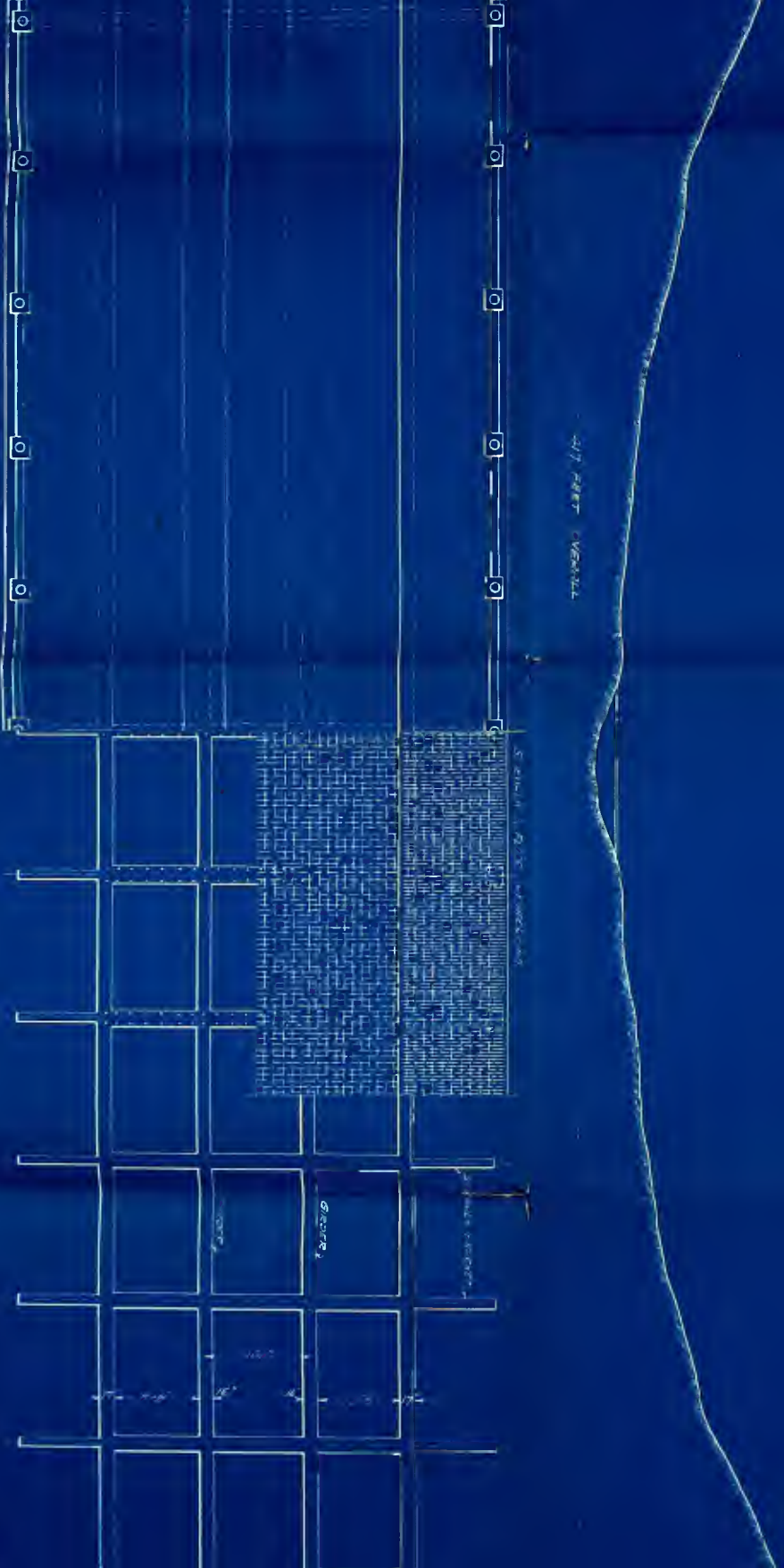
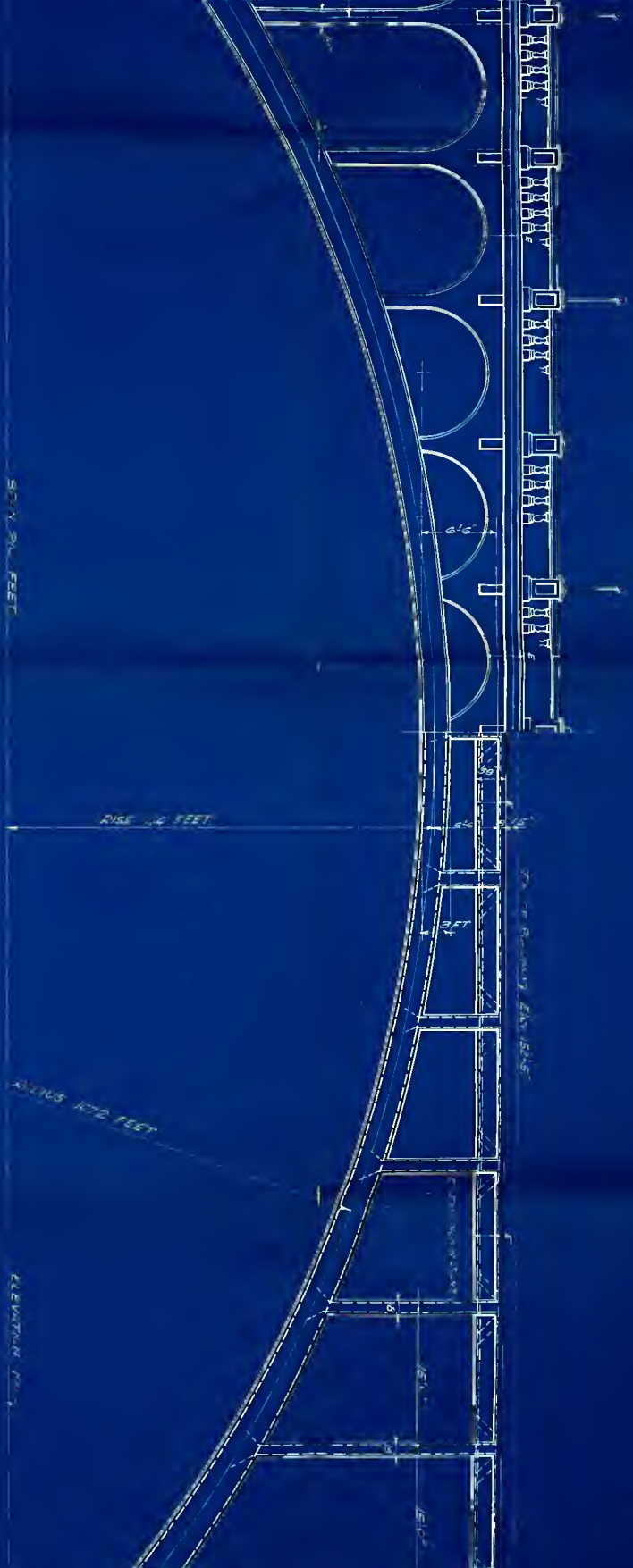
CNE-144LF 1526N

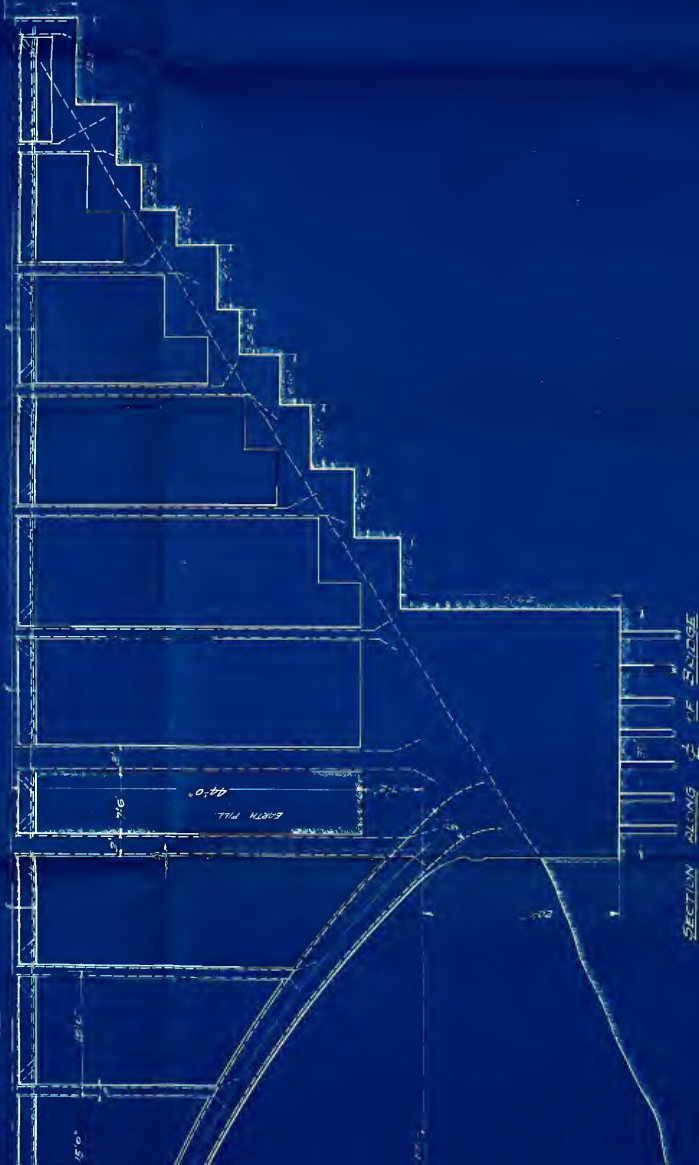
ARMOUR INSTITUTE OF TECHNOLOGY
CIVIL ENGINEERING DEPARTMENT
THESIS

OPEN SPANDREL REINFORCED CONCRETE ARCH BRIDGE
SPAN 210 FEET

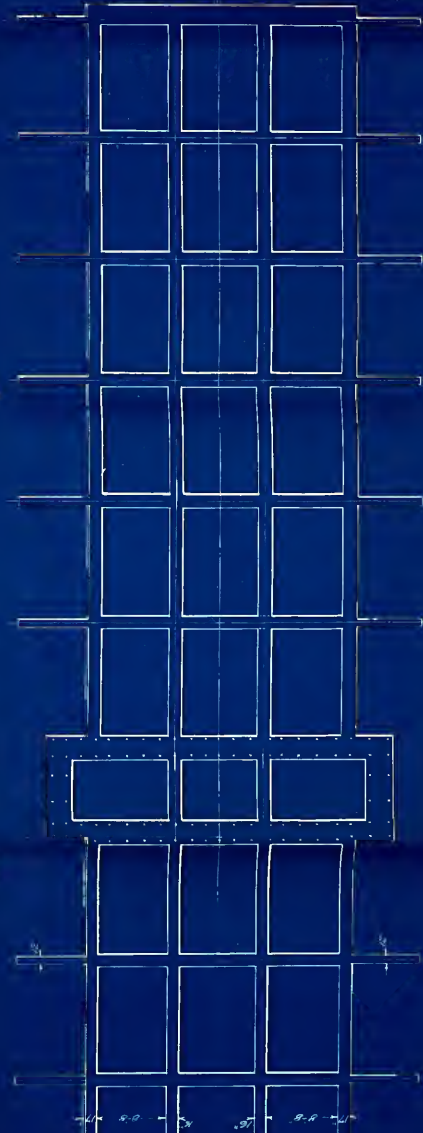
SCALE $\frac{1}{8}$ INCH = 1 FOOT

MAY 1911



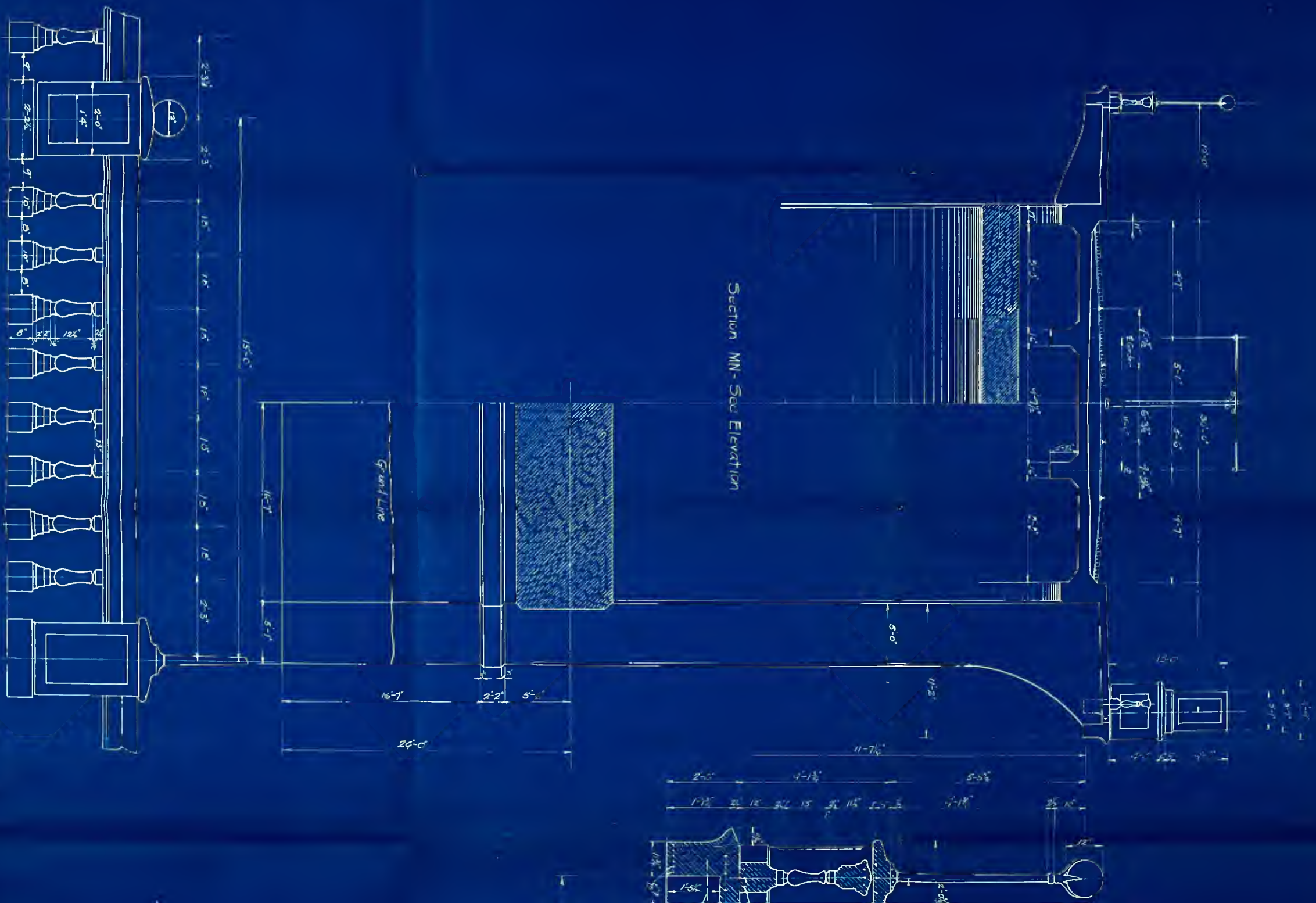


SECTION ALONG C-C OF BRIDGE



SECTION JUST SENEETH FLOOR

Prepared by
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THIS IS

OPEN SPANDREL REINFORCED CONCRETE ARCH BRIDGE

SPAN-2100

$$\text{SCALES} \begin{cases} X_1 = 1\text{-FOOT} \\ X_4 = 1\text{-FOOT} \end{cases}$$

MAY - 1911

RISE 44-O

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